The Design and Shake Table Testing of a Full-scale 7-storey Reinforced Concrete Cantilever Wall

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ABSTRACT: This paper presents a performance-based seismic design strategy used in the design of a 7-storey reinforced concrete cantilever wall, built at full scale, tested on the George E. Brown Jr. Network for Earthquake Engineering Simulation Large Outdoor High-Performance Shake Table at the University of California, San Diego. The design resulted in significant reduction of longitudinal reinforcement in the wall as compared to the current (force-based) code requirements. The test structure was subjected to four historical California input ground motions, including the strong intensity near-fault Sylmar record, which induced significant nonlinear response. Key tests results are presented in the paper. The paper also discusses the results of a blind prediction contest.
1 INTRODUCTION

The new George E. Brown Jr. Large High-Performance Shake Table of the University of California at San Diego enables the dynamic testing of structural systems built at full-scale. In one such large-scale test, structural wall 7-storeys tall, designed using a performance-based strategy, was built and subjected to various input ground motions, including the strong intensity near-fault Sylmar record, which induced significant nonlinear response. This paper discusses the performance-based seismic design strategy and shows key results obtained the test program.

The test structure represented a slice of a 7-storey multistorey residential load bearing wall building prototype located in Los Angeles, California (Englekirk 2007). The test structure is referred to as “the building” hereafter in this paper. Figure 1 shows an overall view of the building and its main components. The lateral force resistance in the building was provided by a 3.66 m long load bearing reinforced concrete rectangular wall, referred to as the web wall. The web wall, which was 0.20 m thick at the first and seventh levels and 0.15 m elsewhere, provided lateral force resistance in the east-west direction and supported seven 0.20 m thick slabs spaced at 2.74 m. Two transverse walls built east and west of the web wall provided lateral and torsional stability to the building. The slab between the web and flange walls was 635 mm wide by 203 mm deep, and at the east and west ends it had a 140 mm deep by 51 mm wide slots. With this geometry, the slab acted like a near-pinned link enabling the transfer of in-plane forces and reduced out-of-plane forces. A 635 mm wide vertical gap between the web and flange walls was left to avoid shear transfer between the web and flange walls.

Figure 1. View of building

The precast segmental wall was connected to each of the slabs with a horizontal steel truss with proprietary low-friction ball bearing connections. The north and south ends of the slabs were supported on four gravity columns consisting of 102 mm diameter extra strong steel pipes filled with high-performance grout to bond a concentric high-strength steel threaded bar that was used to form the column’s end connections. The gravity columns were pinned at the ends and were able to carry axial tension and compression. Panagiotou et al. (2007a) presents a complete discussion of the reinforcing details and construction of the building. The walls and slabs were built using tunnel form construction. So, horizontal construction joints in the wall were located at the top of the slabs and again at 102 mm above the top of the slabs. Concrete with specified compressive strength of 27.6 MPa and ASTM A615 Grade 60 steel reinforcement were used throughout.

2 SEISMIC DESIGN

2.1 Hazard Levels and Performance Objectives

Design lateral forces for the building were obtained for a site in downtown Los Angeles using two different methods. Englekirk (2007) discusses the first method. The second method presented below, addresses the performance of the building at two seismic hazard levels for a site in Los Angeles:

(i) Immediate Occupancy (IO) corresponds to a seismic hazard level associated with frequently occurring earthquakes, of low intensity, with 50% probability in exceedance in 50 years (return period 72 years). The displacement and acceleration design spectra for this hazard level and for 5 percent
viscous damping is shown in Fig. 2. Note that the displacement spectrum is linear in the period range exceeding 0.48 sec., where the spectral velocity is constant. The limit states selected for the building performance at IO are such that no visual damage occurs in the prototype building. This is translated into the following strains and interstorey drift limits: (i) maximum tensile strain of 1 percent in the wall longitudinal reinforcement; (ii) compressive strain of 0.4 percent in the concrete and (iii) interstorey drift ratio of 1 percent. The interstorey drift ratio limit is set to control damage to non-structural elements.

(ii) Life Safety (LS) corresponds to a seismic hazard level associated with the design basis earthquake (DBE) (ASCE-7 2006). The displacement and acceleration design spectra for this hazard level and for 5 percent viscous damping is shown in Fig. 2. Extensive yielding and nonlinear inelastic response is anticipated for the building walls responding to strong intensity earthquakes; therefore, structural damage is accepted in critical regions in the building, in this case at the base of the critical walls. Limit states associated with expected building performance at LS are: (i) maximum tensile strain of 5 percent in the longitudinal reinforcement; and (ii) maximum roof drift ratio of 3 percent. Note that in this procedure, the concrete compressive strain is not considered a limiting strain for the design of walls. Instead, this strain is calculated from the curvature demand required by the governing limit state. Transverse reinforcement is provided in the potential plastic hinge regions of the walls to ensure the compressive strain demand is met. Maximum tensile strain limit is set to avoid fracture of the longitudinal reinforcement while the maximum roof drift ratio limit is used arbitrarily to restrict large nonlinear geometrical (P-Delta) effects.

![Figure 2 – Design spectra and ground motion characteristics.](image)

### 2.2 Mechanism of Inelastic Deformation

The preferred mechanism of inelastic deformation in cantilever walls is through the development of plastic hinges at their bases. Capacity design is used to ensure that this mechanism develops and is maintained throughout (Paulay and Priestley 1994). Other plastic mechanisms are deliberately precluded from developing through the design strength hierarchy. This requires adequate flexural strength above the potential plastic hinge regions, adequate shear strength and development of the longitudinal reinforcement throughout the entire length of the wall.
2.3 First Mode Design Lateral Forces

The design bending moments at the base of the walls are determined using lateral forces derived from the first mode of response. Using code-prescribed load combinations and strength reduction factors, the longitudinal reinforcement in the walls at their bases is established. Because of space limitations, we have not defined all the variables in the equations that follow. The reader is referred to the notation at the end of this paper. For cantilever wall buildings the shape of the first mode is approximately represented by the following polynomial expression:

\[
\Phi_{i,j} = \frac{1}{11} \left( \frac{h_i}{h_n} \right)^5 - \frac{10}{11} \left( \frac{h_i}{h_n} \right)^3 + \frac{20}{11} \left( \frac{h_i}{h_n} \right)^2
\]  

This mode shape is obtained from the deformed shape of a prismatic cantilever wall subjected to distributed lateral forces whose magnitude are directly proportional to the height. The mode shape allows the computation of the first mode modal weight, participation factor and contribution factor,

\[
W_{\Phi} = \frac{\sum_{i=1}^{n} W_i \Phi_i}{\sum_{i=1}^{n} \Phi_i} \quad \text{and} \quad \Gamma_{i} = \Gamma_{i} \Phi_{i,n}
\]  

The target roof displacements are calculated for the limit states governing the IO and LS performance objectives, respectively. For IO, the two strain limit states are met by conservatively assuming the roof displacement in the critical wall, the longest in a regular building (Paulay and Restrepo 1998), does not exceed the yield roof displacement. This assumption has the advantage of not needing to choose a value for the plastic hinge length. Note that for such moderate plastic strain values the plastic hinge is spreading and has not necessarily reached its maximum value (Hines et al. 2004). The yield displacement for the prismatic cantilever examined to obtain the mode shape is:

\[
\delta_y = \frac{11}{40} \frac{\lambda_y}{\ell_y} = \frac{11}{40} \frac{\lambda_y}{\ell_y} h_n^2
\]  

where \( \lambda_y = \frac{1}{\ell_y} \) is the idealized yield curvature as defined in Priestley et al. (2007). When the yield displacement is reached and the building’s lateral displacement is compatible with the first mode shape presented above, the maximum interstorey drift ratio is:

\[
\theta_{y,r} = \frac{15}{11} \frac{\delta_{y,r}}{h_n} = \frac{3}{8} \frac{\lambda_y}{\ell_y} h_n
\]  

The roof displacement \( \delta_{LS,r} \) calculated for LS considers the elastic and plastic contributions. This displacement is given by:

\[
\delta_{LS,r} = M_{LS} \delta_{y,r} + \delta_{p,r} = \left\{ \frac{11}{40} \frac{M_{LS}}{M_n} \frac{h_n}{\ell_y} + \left( \mu_{p,LS} - 1 \right) \left( 1 - \frac{\lambda_p}{2h_n} \frac{\ell_p}{\ell_y} \right) \lambda_y \right\} \lambda_y h_n
\]  

or

\[
\delta_{LS,r} \approx \left\{ \frac{11}{40} \frac{h_n}{\ell_y} + \left( \mu_{p,LS} - 1 \right) \frac{\ell_p}{\ell_y} \right\} \lambda_y h_n
\]
where the LS curvature ductility \( \mu_{\phi,LS} = \frac{\rho_{LS}}{\phi_y} \) in the critical section of the wall is calculated using the curvature \( \rho_{LS} \) corresponding to the governing LS limit state. In Eq. 5(a) the bending moment ratio \( \frac{M_{LS}}{M_n} \) has a small influence in the value of \( \delta_{LS,r} \) and can be assumed equal to unity for all practical purposes. Also, ratio \( \frac{\ell_p}{h_n} \) becomes negligible when \( \frac{h_p}{\ell_w} > 4 \). These simplifications lead to Eq. 5(b) shown above. The roof drift ratio at LS, is calculated by dividing both sides of Eq. 5(a) or (b) by \( h_n \), that is, \( \theta_{y,r} = \frac{\delta_{LS,r}}{h_n} \).

The following step in the method requires converting of the limiting IO and LS roof displacements into equivalent displacements, \( \delta_{IO,e} \) and \( \delta_{LS,e} \), respectively, of linear single-degree-of-freedom oscillators:

\[
\delta_{IO,e} = \frac{\delta_{y,r}}{T_I} \quad \text{and} \quad \delta_{LS,e} = \frac{\delta_{LS,r}}{T_L} C_\mu
\]  

The definition of \( \delta_{IO,e} \) is taken from Chopra (2001). The definition for \( \delta_{LS,e} \) is based on empirical relationships relating the displacement responses of inelastic to elastic oscillators of equal initial periods (Chopra 2001; Ruiz-Garcia and Miranda 2003). A specific study of the 360° component Sylmar ground motion obtained during the 1994 Northridge, California, earthquake gives values of \( C_\mu \) between 0.7 and 1.2 in the period range of 1 to 2.2 sec for \( \mu_{\phi} = 6 \). This motion was used in the experimental program to represent the design basis earthquake. The maximum value of \( C_\mu = 1.2 \) was adopted for the design. The relatively small value of \( C_\mu \) for the specific ground motion is due to the fact that the predominant periods of the distinct pulses contained in this motion are smaller than the period of the building (Ruiz-Garcia and Miranda 2006; Panagiotou 2008). The equivalent displacements calculated from Eq. 6 are \( \delta_{IO,e} = 64 \text{ mm} \) and \( \delta_{LS,e} = 323 \text{ mm} \). The maximum periods \( T_{IO} \) and \( T_{LS} \) corresponding to the critical limit states at IO and LS, respectively, are computed using the equivalent displacements and the slope of the linear portion of the displacement spectra. This procedure gives in \( T_{IO} = 1.06 \text{ sec} \) and \( T_{LS} = 1.53 \text{ sec} \).

The smallest of the periods obtained indicates which performance level controls the design of the building. In the design of the building, IO governs. In the final design, the fundamental period of the building, calculated ignoring tension stiffening, must be such that \( T_I \leq T_D \), where \( T_D = \min(T_{IO}, T_{LS}) \). When \( T_{LS} > T_{IO} \), the LS strain limit may be revised iteratively until \( T_{LS} \approx T_{IO} \). This allows relaxation of the reinforcing detailing in the potential plastic hinge regions. For example, when the longitudinal reinforcement LS tensile strain limit is revised to 3 percent, then \( \mu_{\phi,LS} = 10 \), \( \delta_{LS,r} = 344 \text{ mm} \), \( \delta_{LS,e} = 236 \text{ mm} \), and \( T_{LS} = 1.12 \text{ sec} \). Once the period \( T_D \) is established, the first mode design base shear, \( V_{b,1} \), is calculated as follows:

\[
V_{b,1} = \frac{\delta_{y,r}}{T_I} \left( \frac{2\pi}{T_D} \right)^2 \frac{W_{el}}{g}
\]  

(7)
For the building, Eq. 7 gives $V_{h,1} = 292 \, kN$. The design base shear coefficient, as defined by ASCE-7 (2006) is $C_v = 0.15$. The first mode base shear is distributed in lateral forces $F_{i,j}$ in proportion to $W_i \Phi_{i,j}$ (Chopra 2001). The design bending moment at the wall base, $M_u$, computed by taking moments of the lateral forces about the wall base, is $M_u = 4243 \, kN-m$.

### 2.4 System Static Overstrength Lateral Forces

The system static overstrength lateral forces (SSOLF) are those lateral forces, greater than the design lateral forces, required to push the critical building wall (with its actual boundary conditions and longitudinal reinforcement specified at the critical sections) to the governing LS limit state. This analysis used expected rather than specified material properties. In practice, the SSOLF could be determined using an adaptive pushover analysis of a representative mathematical model built in a nonlinear analysis software (Reinhorn 1997; Satyarno et al. 1998). In the design of the building it is useful to estimate these forces as the addition of two independent and additive mechanisms: (i) an element mechanism that assesses the effects of flexural overstrength at the critical section of the wall; and (ii) a system mechanism that assesses the effects caused by kinematics on the slabs framing onto the laterally deformed wall. These two mechanisms are examined below.

#### 2.4.1 Element Mechanism

Excess in the flexural strength in the plastic hinge of a cantilever wall is caused by excess longitudinal reinforcement, expected rather than specified material properties, hardening in the longitudinal reinforcement and larger axial compression in the wall. When these factors are considered, the flexural capacity at the critical section at LS is such that $M_{LS} > M_u$, the flexural overstrength factor is defined as $\Omega_{LS,j} = \frac{M_{LS,j}}{M_u}$. To attain the LS limit state on the bare cantilever wall, the first mode lateral forces must be increased to $\Omega_{LS,j} F_{ij}$. Factor $\Omega_{LS,j}$ may be computed from moment-curvature analyses of the critical section of the wall, as detailed, for each load combination, $j$. These analyses are performed using expected material properties. Because of the relatively low axial compression force levels that nearly all building cantilever walls experience, demands for combined flexure-axial load design most often governed by the flexural overstrength factor found for the load combination resulting in the minimum axial force in the wall. For the revised LS tensile strain limit of 3 percent, the flexural overstrength factor for the single load combination is $\Omega_{LS} = 1.44$.

#### 2.4.2 System Mechanism

In 1985, Bertero et al. examined comprehensively the increase in lateral resistance caused by deformation compatibility between walls and gravity load elements framing into them. This interaction causes large increase in the shear force demand in walls as well as an increase in the system moment capacity. Because such forces arise from deformation compatibility, this source of additional strength is referred to here as kinematic overstrength. The initial design of the building ignored this mechanism completely, however, since this mechanism had a large effect on the building’s response, it was included as part of the design in this paper.
Figure 3 depicts a two-dimensional deformed state sketch of the building at its peak westward displacement. The migration of the neutral axis depth towards the extreme compressive fibre and corresponding development of large strains in the tensile reinforcement in the plastic hinge region of the wall causes significant elongation of the tension chord of the wall. For example, for the revised LS tensile strain limit of 3 percent in the extreme longitudinal bar in tension, the expected tensile chord elongation at the first level of the web wall is 47 mm. The accumulated elongation in levels 2 to 7 is expected to only slightly increase above 47 mm because of anticipated elastic response of the web in these levels. Although ignored in the initial design of the building, this elongation is sufficiently large enough to mobilize a local mechanism in the slotted slab. The wall tensile chord lengthening causes slab end rotations of at least 0.077 radians. At such a large rotation, the slab develops its small flexural capacity at the slotted ends over the entire 4.88 m length of the slots at each floor.

The remaining twenty percent is carried by the gravity columns. The additional web wall base compressive force adds to the force needed for equilibrium in the flexural overstrength mechanism. Initially it seems counterintuitive that there is no tensile force resisting moment at the base of the wall. This is because the tensile force in the reinforcement has already been considered in its entirety to resist bending in the flexural overstrength mechanism and these two mechanisms are considered independent and additive. Furthermore, for compatibility, the eccentricity between the compressive forces in the critical section in the two mechanisms must be the same, requiring an iterative process; therefore, the superimposition of the two mechanisms should be considered as a first order approximation.

Swaying of the wall also induces combined three-dimensional bending in the slabs. This is because the gravity columns that are located at the slab north and south edges restrain the slab from rotating, thus enabling the development of a moment \( M_w \) at each level. A nonlinear finite element analysis of the slab gave a moment capacity of \( M_w = 204 \text{kN-m} \), this moment is resisted by a tension-compression pair by the gravity columns.

In the undeformed state, the lateral forces \( F_{i,j} \) needed to form the mechanism shown in Fig. 3 are given by:

\[
F_{i,j} = \frac{M_w + Q_i \left( e_i + a + \frac{\varepsilon_w}{2} \right)}{h_j}
\]  

Eq. 8 ignores the additional, but small, lateral forces carried by the flange wall. These lateral forces take the form of a harmonic series, with the force in the lowest level being the greatest. In multistorey buildings the height of these resultant lateral forces is generally low. In the case of the building, it is located at 39 percent of the roof height. Compared with the first mode lateral forces \( F_{1,j} \), the forces \( F_{i,j} \) are not negligible. In fact, it is quite remarkable that despite the sum of slab moments on all seven levels is \( 7 \left[ M_w + Q_k \left( a + \frac{\varepsilon_w}{2} \right) \right] = 0.73 M_w \), the base shear ratio is \( V_{k,w} / V_{k,3} = 2.07 \). Such a large
ratio suggests that kinematic effects induced by slabs and/or gravity load beams framing into cantilever walls should be considered in design, even if only in an approximated way, or accounted for with a pushover analysis.

2.5 Higher Mode Effects

Parametric analyses performed by Panagiotou (2008) suggest that in most wall buildings, including regular tall buildings, the effects of the higher modes of response are largely dominated by the second translational mode. For the design of the building, the second mode is approximated by the following cubic polynomial:

\[ \phi_{2i} = 2.4 \left( \frac{h_i}{h_o} \right)^3 - 8.6 \left( \frac{h_i}{h_o} \right)^2 + 5.2 \frac{h_i}{h_o} \]  

The second mode modal weight, \( W_{e,2} \), participation factor, \( \Gamma_{2} \), and contribution factor, \( \Gamma_{2}^* \), are calculated using the generalized form of Eq. 2. For the building, \( W_{e,2} = 0.17 W_i \), and \( \Gamma_{2}^* = 0.62 \). Because the first-mode period and the structural system type are known, the second mode period can be approximated. For prismatic Euler-Bernoulli cantilever beams of uniform distributed mass, the second mode is \( T_2 = \frac{T_i}{6.3} \). The DBE spectral acceleration, corresponding to \( T_2 \) is available from the design spectra. Thus, the second-mode base shear is:

\[ V_{b,2} = \frac{S_u}{g} W_{e,2} \]  

For the test structure, for the building, \( S_u = 1.81g \), see Fig. 3, and \( V_{b,2} = 615 \text{kN} \), which is equal to \( 2.1 V_{b,1} \).

2.6 Combinations

At every storey level, system overturning, shear forces and axial forces are computed from the two mechanisms evaluated before and from the second mode, and then combined using the square root sum of the squares (SRSS) rule to obtain design actions as follows:

\[ U_i = (1.0 U_{D,i} + 1.0 U_{L,i}) \pm 1.0 \sqrt{\left( \Omega_{LS} U_{1,i} + U_{K,i} \right)^2 + (U_{2,i})^2} \]  

For the web wall bending moment, Eq. 11 is used where \( i = 2 \) to 7. This is because at the base of the web wall the design moment is established previously. Note that in Eq. 11, the forces of the element and system mechanisms are added before squaring them because they develop concurrently.

3 WALL DESIGN

The longitudinal reinforcement in the potential plastic hinge region of the web wall is calculated from the first mode base moment \( M_u = 4243 \text{kN-m} \) and the gravity load axial force \( N_u = 809 \text{kN} \). With specified material properties \( f_y = 414 \text{MPa} \) and \( f_y' = 30 \text{MPa} \), the required longitudinal reinforcement ratio for the \( (M_u, N_u) \) pair is \( \rho_i = 0.59\% \). Figures 8 and 9 show relevant reinforcing details for the web wall. The longitudinal reinforcement in this region of the web wall consisted of
eight #5 (16 mm) bars in the boundary elements. In between boundary elements, thirteen #4 (13 mm) longitudinal bars were detailed in a single curtain (see Fig. 4). The reinforcement ratio in the first level of the web wall was $\rho = 0.66\%$.

4 TEST PROGRAMME

4.1 Input Ground Motions

The building was subjected to four historical earthquakes, of increasing intensity, recorded in southern California. Prior to and between earthquake shake table tests the building was subjected to long-duration (8 min) ambient vibration tests and to long-duration (3-min) low-amplitude white noise (WN) excitation tests. The motion of the shake table during the WN tests consisted of 0.5-25Hz band clipped WN acceleration processes with root-mean-square (RMS) amplitudes of 0.02, 0.03 and 0.05g. The 0.03g RMS WN tests excited the web wall beyond cracking, but within the elastic limit of the reinforcement. These tests were used for system identification (Moaveni et al. 2009) and to evaluate damage progression in the building (He et al. 2009).

The acceleration time-histories as well as the elastic 5% damped acceleration and displacement response spectra of the earthquake input motions, as reproduced by the LHPOST, are shown in Fig. 2 above. This figure also depicts the target response spectra for immediate occupancy and for the ASCE-7 (2006) design basis earthquake. It also contains the response spectra for the 3 minute long 0.03g RMS WN table motion whose intensity was low relative to the earthquake motions. The lowest intensity input motion EQ1 consisted of the longitudinal component from the VNUY station recorded during the 1971 $M_w$ 6.6 San Fernando earthquake. The two medium intensity input motions EQ2 and EQ3 were taken as the transverse component recorded at the VNUY station obtained during the 1971 $M_w$ 6.6 San Fernando earthquake and the longitudinal component from the WHOX station recorded during the 1994 $M_w$ 6.7 Northridge earthquake, respectively. The large intensity input motion EQ4 corresponded to the Sylmar Olive View Med 360° recorded during the 1994 $M_w$ 6.7 Northridge earthquake.
4.2 Test Results

4.2.1 Dominant Periods

A fundamental period of $T = 0.51$ sec was obtained from the 0.03g RMS WN tests at the beginning of the test program. It is close to the theoretical fundamental period of 0.50 sec obtained from a three-dimensional model of the building using uncracked section properties and accepted elastic properties for concrete. The periods cited here were identified from 0.03g RMS WN tests. Before performing test EQ1, the fundamental period of the building had shifted to $T = 0.59$ sec. This was due to the partial loss of tension stiffening in the concrete caused by twenty five 0.02 and 0.03g RMS WN tests performed before EQ1. After test EQ1, the fundamental period shifted to $T = 0.65$ sec. After tests EQ2 and EQ3, the fundamental period increased to 0.82 sec and 0.88 sec, respectively. The fundamental period lengthening was the result of the gradual loss of tension stiffening across the cracked concrete. Finally, after test EQ4, the fundamental period reached $T = 1.16$ sec. In contrast with the first mode, the second mode period, obtained for low amplitude vibration, only slightly changed and remained close to $T_2 = 0.1$ sec.

4.2.2 Drift Ratios

Test EQ1 deformed the building to a maximum roof drift ratio, defined as the ratio between the maximum lateral displacement at the uppermost level and the distance of this level to the base of the wall, equal to $\Theta_r = 0.28\%$. The maximum recorded interstorey drift ratio was 0.35% or 1.25 $\Theta_r$. The building response parameters for tests EQ2 and EQ3, respectively, were similar, although some subtle differences could be observed, especially in those response parameters sensitive to high frequency content of the excitation. The peak roof drift ratios measured in these tests were $\Theta_r = 0.75$ and 0.83%, respectively. Recorded interstorey drift ratios in these tests were 1.19 and 1.24 of their respective roof drift ratios. At the base of the wall, moderate yielding occurred in the web walls longitudinal reinforcement, which reached a tensile strain of 1.73% and 1.78%, respectively. The concrete compressive strain measured near the extreme compressive fibre at the base of the wall reached -0.17% and -0.18% in tests EQ2 and EQ3, respectively. Yielding took place also in the extreme longitudinal reinforcement of the web wall at the base of level 2 in tests EQ2 and EQ3. Measured tensile strains at these two tests in level 2 reached 0.36 and 0.39% on the west and east sides of the wall, respectively. In both tests, tensile strains in the extreme longitudinal reinforcement in level 3 were very close to the yield strain. In these tests the maximum shear deformation along the construction joints was 0.4 and 0.5 mm, respectively. The roof residual displacements were 4.3 mm in both tests EQ2 and EQ3. The highest strain rate in the longitudinal reinforcement was measured during test EQ2 and reached 40%/sec. This was caused by the first spread of the Lüders bands into the gauged portion of the bars during this test. This level of strain rate resulted in a 7% increase of the steel yield strength during test EQ2 according to coupons tested under strain controlled conditions in a universal testing machine to the same strain rate and strain history (Panagiotou et al. 2007b).

In test EQ4, the maximum roof drift and interstorey drift ratios were 2.06% and 2.36%, respectively, with a maximum interstorey drift ratio of 1.15$\Theta_r$. A comparison of the ratios between the maximum interstorey drift and roof drift ratios in tests EQ1 to EQ4 shows a consistent reduction with the progression of testing. This is also manifested in the development of localized plasticity at the base of the wall, which increased as the displacement demand in the tests increased. In this test, the tensile strain in the longitudinal reinforcement in the plastic hinge at the base of the wall reached 2.85% and the concrete strain near the extreme compressive fibre reached -0.39%. This strain level is in the accepted range of strains; spalling of the concrete cover was observed. The weld resistance confinement grids provided excellent lateral stability to the reinforcing bars once the concrete cover spalled off since no evidence of longitudinal bar buckling was observed at the end of Phase I testing. Significant yielding in the longitudinal reinforcing bars occurred at level 2, with the maximum measured tensile strain to reach 0.88 and 1.95% on the east and west side of the web wall, respectively. This is in general agreement with the analysis9) which predicted that both sides of the wall at level 2, and especially the west side, would experience yielding. Levels 3 and 4 yielded also solely on the west side, with the maximum tensile strain to reach 0.25 and 0.28%, respectively. This
also verifies the analysis results that indicated the possibility of yielding on the west side of the wall at level 4.

4.2.3 Hysteretic Response

Figure 5 shows the system base overturning moment and base shear versus roof lateral relative displacement hysteretic responses measured for tests EQ1 through EQ4. Positive displacement is defined towards west.

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The building system base overturning moment was estimated as the sum over all floors of the product of the tributary seismic mass, the total floor acceleration and the floor height over the base of the wall. The base overturning moment also accounts for the small P-Delta effects as well as for the small mass...
rotatory inertia effects. The total floor acceleration was calculated as the average of the three horizontal accelerometers at each floor. Lateral displacements were calculated from the accelerations measured by the three horizontal accelerometers at every floor using filtering and double integration of the measured acceleration. A high-order (5000), 0.2-25 Hz band-pass, FIR filter was used in Matlab (Matlab 2008). The calculated displacement time histories were in excellent agreement with those measured directly with the GPS displacement sensors in the test program (Bock et al. 2006, Panagiotou et al. 2007b). Figure 5a also shows the web wall design base moment, $M_w$, as well as the maximum system base moments $M_{SE}$ and $M_{SW}$ for east and westward response, respectively, calculated with the design method used. Very good agreement is observed between the estimated and measured system base overturning moment for the westwards response. Note that the method of analysis did not consider the small bending moment carried by the flange wall and did not account for the strain aging effects that played a role for eastward response. Strain aging occurs in certain types of steel after undergoing plastic deformations when soluble Nitrogen diffuses through free dislocations in the steel structure and pins them, changing its mechanical properties and rising the yield strength upon time (Restrepo-Posada et al. 1994). The lapse between tests EQ3 and EQ4 was 55 days, sufficient for the development of some strain aging. Rebar test coupons were subjected to the same strain rate and strain history, and were tested with and without the lapse between tests for EQ3 and EQ4 (Panagiotou et al. 2007b). These tests indicate that strain aging contributed to an increase of 7% of the flexural strength of the web wall for eastward response.

The system total base shear force versus roof lateral displacement hysteretic response is plotted in Fig. 5b. This figure also shows the design base shear force, $V_w$, and the east and westward system base shear forces, $V_{SE}$, $V_{SW}$, including section and kinematic system overstrength as well as second mode effects computed from the displacement-based design method (Panagiotou and Restrepo 2009). Excellent agreement is observed between maximum measured and predicted base shear forces for both directions of the response. Note that such good agreement is merely a coincidence as will be explained later in section Response Envelopes.

Comparison of the system base overturning moment and total base shear force hysteretic responses shows: (i) significant system overstrength and (ii) erratic loop traces in the base total shear force hysteretic response when compared to the loops observed from the base moment hysteretic response. The base moment overstrength factor calculated as the ratio of the maximum measured system base overturning moment and design base overturning moment of the web wall only was $\Omega_{OM} = 2.71$. The observed system base moment overstrength is due to: (i) section flexural overstrength, and (ii) kinematic system overstrength. These two sources of overstrength will be examined in detail in the following sections. The base shear overstrength factor calculated as the ratio of the maximum measured system base shear force and the design base shear force of the web wall only was $\Omega_{OV} = 4.20$. The difference in magnitude between $\Omega_{OM}$ and $\Omega_{OV}$ is due to the higher modes in the response of the building. The influence of the higher modes will also be discussed below. Analysis of the data conclusively showed the wall responded primarily in flexure as intended.

A source of kinematic overstrength in the building was the warping and bending of the slabs caused by the deformation of the web wall (i.e., lengthening of tensile chord and shortening of compressive chord). The slabs warped and bent because of the restraints imposed by the gravity columns. During test EQ4, the tensile chord lengthened by 65 mm over the entire height of the wall. This growth caused significant tension and additional compression forces in the gravity columns. In this test the maximum moment resisted by the first level gravity columns, which had been instrumented with strain gages, was 12% of the maximum measured system base overturning moment and 33% of the web wall design moment $M_d$. Another source of kinematic overstrength was the slotted slabs between the web and flange walls. Elongation and shortening of the east chord of the web wall occurred for westwards and eastward displacement response, respectively. Such change in length was negligible in the flange wall due to the small level of strains developed. Thus, each slotted slab was forced to rotate and develop positive and negative yield lines along the slots.
5 PREDICTION OF THE RESPONSE

5.1 Model Uncertainty: Blind Prediction Contest

The arrival in recent years of high-performance multiprocessor personal computers, large storage capacity and of advanced nonlinear analysis tools has made it possible for engineers to make use of these tools in routine work. A blind prediction contest organized to compare analytical with measured responses had worldwide entries with reasonably accurate predictions. However, a large percentage of the entries submitted fair predictions. The overall scatter in the predictions increased with inelastic response and was greater for force and acceleration responses as was for displacements. The lesson learned from the blind prediction is that engineers should become aware of the effects of model uncertainty in their decision-making process.

Different types of models, from linear static to nonlinear dynamic, developed from the different participants. The prediction quantities of the contest were the system bending moment, shear force, lateral relative displacement, floor total acceleration and floor interstorey drift ratio envelopes. Figure 6 plots the response envelopes and the predictions of the eight best teams of the contest. The response was predicted better in terms of the lateral displacements and interstorey drift ratios. Underestimation of the system bending moment and shear force observed. This was mainly due to the inadequate modeling of the flexural framing between the walls, the slab and the gravity columns which resulted in significant increase of the system forces. Important issue of the modeling was also the larger values of viscous damping used in comparison with the viscous damping estimated from the authors for the verification of the experimental response.

Figure 6- Results of blind prediction contest
In summary, the blind prediction exercise proved to be an excellent platform to gage model uncertainty. The ability to predict the response reasonably with only limited amount of time suggests excellent progress in the field. However, many entries submitted fair predictions we believe this was chiefly because of the lack of understanding of the mechanisms that contribute to overstrength. Such lack of accuracy results in model uncertainty. Engineers should account for such uncertainty in the decision-making process when using advanced tools in structural analysis.

5.2 Structural Viscous Damping

There is little agreement for the use of damping ratios for conducting nonlinear time-history analyses. It has been common practice to use damping ratios for the first mode set at 5% of critical. Data extracted from the test and comparison with refined models, indicate that such ratios too high and cannot be justified. A similar observation was made by Petrini et al. (2008), who conducted shake table testing on a bridge column. Small amount of viscous damping was reduced from the building’s inelastic response. Figure 7 plots the comparison of measured and computed response in terms of top lateral relative displacement time history response for EQ2 and EQ4. The calculated response is presented for two values of viscous damping (0.25% and 2% for the first mode). Even for small values of viscous damping of 2% the response is underestimated in comparison with the measured response. A small value of viscous damping of 0.25% estimated for the reproduction the experimental results. Calibration of the computational model showed that as far as all the sources of the hysteretic response have been specified the additional viscous damping for such a system is minimal, especially under extensive nonlinear response. Typically, larger damping values are used in design to account for sources of energy dissipation not considered in the analysis. Since a response can be very sensitive to the damping ratio, the authors recommend the use of interstorey hysteretic damping instead to account for the unknown sources of nonlinear response.

![Figure 7 - Effect of viscous damping, comparison of measured versus computed top displacement response](image-url)
6 CONCLUSIONS

- This test provided clear evidence in support of new performance-based seismic design methods. In the seismic tests of the building, all the performance objectives selected for immediate occupancy and life-safety were met despite the design base shear was about 50% of that required by a prescriptive code.

- Three-dimensional interaction effects between the web wall, flange wall and the slabs referred to here as kinematic overstrength caused significant increase in the system overturning moment capacity as well as of the shear force demand in the web wall. Given the undesirable consequences of shear failures in reinforced concrete buildings, such larger than expected shear force demands should be accounted for in design.

- Dynamic effects (higher mode effects) observed in the response of the building system can augment the shear force demand in individual walls and significantly increase the total accelerations along the height of the building.

- A blind prediction of the response of the test structure indicates that the development of model for nonlinear analyses can incorporate significant uncertainty.

- System identification of the nonlinear response of the test structure confirms that nonlinear analyses should be conducted with low viscous damping ratios.

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